1969

Summary report to rcrbsj by project staff, March 1969

J. W. Fisher

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STUDIES OF BOLTED CONNECTIONS

p14

p11

SUMMARY REPORT TO
THE RESEARCH COUNCIL ON
RIVETED AND BOLTED
STRUCTURAL JOINTS

What are design guides for single joint?

Ls vs L Butt

by

Project Staff

(Not for publication)

This work has been carried out as part of the studies on Bolted Connections sponsored financially by the Pennsylvania Department of Highways, the Louisiana Department of Highways, the Department of Transportation, Bureau of Public Roads, the American Institute of Steel Construction and the Research Council on Riveted and Bolted Structural Joints. Technical guidance is provided by the Research Council on Riveted and Bolted Structural Joints.

Mention M&M

Project Staff: L. S. Beedle, J. W. Fisher, S. K. Desai, U. Rivera

March 1969

Fritz Engineering Laboratory Report No. 340.3

STATUS OF VARIOUS PHASES OF PROJECT 317

"BOLTED HIGH-STRENGTH STEEL JOINTS"

AISC?

Sponsored by Pennsylvania Department of Highways at Lehigh University

	Phase and Topic	Remarks	Tests Performed	Tests to Be Completed	Analytical Work	Reports
I.	Quenched & Tempered Steel (ASTM A514) Joints Fastened With A490 Bolts)	Completed	Pilot Studies - Six Tests. Full size tests 8 large joints	None	Ultimate strength, load distribution, slip behavior	317.8 317.9 317.13
II.	Hybrid Connections Two or more different grades of steel are joined	Completed	12 shear jig tests. HJ131, HJ132 HJ136, HJ133 HJ135	None	Ultimate strength and load distribution studies	317.3 317.5 317.11
III.	Quenched and Tempered Steel Joints Fastened With A325 Bolts	Completed	Tension shear jigs tested, Pilot Tests J42a, J42b, J42c, J42d, F191, F192 F131, F132	None	Ultimate strength and load distribution studies	317.9 317.11 317.13

PROJECT 317

Summary of Reports - to March 1969

317.1 Project Staff
"Summary Report to Committees 10 and 23", September 1965

317.2 Project Staff
"Summary Report to the Research Council on Riveted and Structural Joints", March 1966

317.3 R. Kormanik
"The Behavior of Hybrid Bolted Connections", Master of Science Thesis, July 1966

317.4 Project Staff
"Summary Report to the Research Council on Riveted and Structural Joints"

R. Kormanik and J. W. Fisher
"Bearing-Type Bolted Hybrid Joints", Journal of the Structural Division, ASCE, Vol. 93, ST5, October 1967

317.6 Project Staff
"Summary Report to Committees 10 and 23", December 1966

317.7 Project Staff
"Summary Report to Committees 10 and 23", April 1967

G. L. Kulak and J. W. Fisher
"A514 Steel Joints Fastened by A490 Bolts", Journal of the Structural Division, ASCE, Vol. 94, ST10, October 1968

317.9 G. L. Kulak
"The Analysis of Constructional Alloy Steel Bolted Plate Splices", Ph.D. Dissertation, June 1967

317.10 Project Staff
"Summary Report to Committees 10 and 23", October 1967

J. W. Fisher and G. L. Kulak
"Tests of Bolted Butt Splices", Journal of the Structural Division, ASCE, Vol. 94, ST11, November 1968

317.12 Project Staff
"Summary Report to Committees 10 and 23", March 1967

317.13 G. L. Kulak and J. W. Fisher
"Behavior of Large A514 Steel Bolted Joints", February 1968, Approved for publication in the Journal of the Structural Division, ASCE

STATUS OF VARIOUS PHASES OF PROJECT 318
"SERVICE PERFORMANCE OF BOLTED JOINTS"

Sponsored by Pennsylvania Department of Highways at Lehigh University

Phase and Topic	Remarks	Tests Performed	Tests to Be Completed	Analytical Work	Reports
I. Out-of-Flat Large Joints	Completed				318.5
II. Effects of the Variation of Contact Area on Slip Resistance of Bolted Joints	Completed	All joints blast cleaned and with surface treatment		Effect of the filler plates on slip behavior	318.1 318.6
III. Effect of slotted and oversize holes upon joint behavior	Completed	21 joints with oversize and slotted holes	None	Slip behavior and ultimate strength studies	318.2 318.3

PROJECT 318

Summary of Reports - to October 1967

318.1 E. Nester
"Influence of Variation of the Contact Area Upon Slip Resistance of a Bolted Joint", Master of Science Thesis, July 1966

318.2 R. N. Allan
"The Effect of Oversize and Slotted Holes on the Behavior of a Bolted Joint", Master of Science Thesis, May 1967

R. N. Allan and J. W. Fisher
"Bolted Joints with Oversize or Slotted Holes", Journal of the Structural Division, ASCE, Vol. 94, ST9, September 1968

318.4 J. H. Lee
"The Effect of Rectangular and Circular Fillers on the Behavior of Bolted Joints", Master of Science Thesis, May 1968

318.5 J. H. Lee, C. O'Connor and J. W. Fisher
"Effect of Surface Coatings and Exposure on Slip Behavior of Bolted Joints", July 1968

318.6 J. H. Lee and J. W. Fisher
"Bolted Joints with Rectangular or Circular Fillers", June 1968

STATUS OF VARIOUS PHASES ON PROJECT 340A

"STUDIES OF SIMULATED BRIDGE JOINTS"

Sponsored by Louisiana Department of Highways at Lehigh University

Phase	Topic	Remarks	Tests Performed	Tests to Be Completed	Analytical Work	Reports
I.	Effects of Out-of-flatness	Completed	Calibration of device to evaluate force to flatten plates 5 plate tests & 3 special slip tests	None	Analysis of forces required to flatten plates	317.12
II.	Control Test	Completed	5 control joints, 2 riveted and 3 bolted calibration studies	None		340.1 340.2
III.	Full size simulated Joint Test	Completed	1 riveted joint 1 bolted joint	None	Load partition in shingle joints	340.1 340.2

Work under Phase I and III is being continued under Project 340B

PROJECT 340A

Summary of Reports - to March 1969

340.1 N. Yoshida
"Behavior of Large Shingle Splices that Simulate Bridge Joints", Master of Science Thesis, May 1968

340.2 N. Yoshida and J. W. Fisher
"Large Shingle Splices that Simulate Bridge Joints", December 1968

340.3 Project Staff
"Summary Report to Committees 10 and 23", March 1969

STUDY OF VARIOUS PHASES ON PROJECT 340B

"STUDIES OF SIMULATED BRIDGE JOINTS"

Sponsored by Louisiana Department of Highways at Lehigh University

Phase	Topic	Remarks	Tests	Analytical Work	Reports
I.	Out-of-flatness in large joints	Continuation of work started under 340A			317.12
II.	Retests on large simulated bridge joints	Retesting test joints after modification	Large bolted splice tested on 3/13/69. Large riveted splices to be tested in April 1969	Load distribution in the elastic region	None
III.	Ultimate strength of shingle joints	Test program to be developed		Ultimate strength of shingle joints	

Project 340B - Phase II: Retests on Large Simulated Bridge Joints

Introduction

The tests reported in Fritz Laboratory Report 340.2 on large shingle joints (see Fig. 1) were loaded up to the capacity of the 5,000,000 lb. testing machine. Except for slip, no marked or non-linear behavior was observed in the joints and it was not possible to determine the joint strength. Therefore, it was decided to reduce the net cross sectional area of the joints so that failure could occur within the machine capacity. Since the tensile strength of the plate material was 88 ksi, the net area of 101.6 in^2 had to be reduced.

Three major factors were considered when developing the joint modifications:

(1) The results of the modified joint test were to be correlated with the test results of the original joint. Therefore, it was important that the ratio of the net plate area to the fastener shear area be maintained.

(2) It was desirable to modify the joint without disassembling it in the test portion. Major slippage had already occurred throughout the joint length and the fasteners were leaning bearing against the plates. The intent of subsequent testing was to continue the loading until failure in order to observe joint behavior and strength.

(3) The joint should fail in the test portion and not in the loading grips. The grip areas of the joints had to be reinforced because some of the plates in this area had cracked during the earlier test and most of the rivets had sheared off.

A sketch of the modified joint is shown in Figure 2. Figure 3 shows the joint shear planes and the net areas in the original and the modified joints.

Fabrication

The fasteners in the grip areas were removed in both joints so that additional plates could be added. None of the fasteners in the test portion were disturbed. A514 and A573 high strength steel plates were added at both ends of each joint and drilled to match the existing hole patterns. The original drilled holes were distorted due to the prior loading, and it was necessary to ream the resulting plate assembly so that 1 in. A490 bolts could be installed.

The area in the test portion was reduced by removing the existing angles and reducing the width of the joint plates. In doing this, two lines of fasteners in each side were removed which reduced the shear area. The reduction in plate area was accomplished while the shingle joints were still assembled. A line of holes was drilled through all plates along each edge of the joints. The remaining plate area was then cut off with an acetylene torch. The rough finish was then milled until the desired reduction in area was obtained. All surfaces were milled until smooth to provide a uniform width and remove any damaged material.

FUTURE WORK

The ultimate strength of the modified shingle joints will be determined by testing to failure in the 5,000,000 lb. machine. As in the previous tests, local slip behavior, total joint elongation and the force-distribution along the joints will be obtained. Based on preliminary estimates, the expected failures in the joint are: plate failure in the bolted joint anything? and rivet failure in the riveted joint.

A mathematical model for partition of load in the test joint has been developed for the elastic range. Work has begun on the comparison of the theoretical solution with experimental results. The tests results from the proposed tests will also be compared to the theoretical solution and attempts will be made to extend the solution into the non-linear region.

model was not a math model

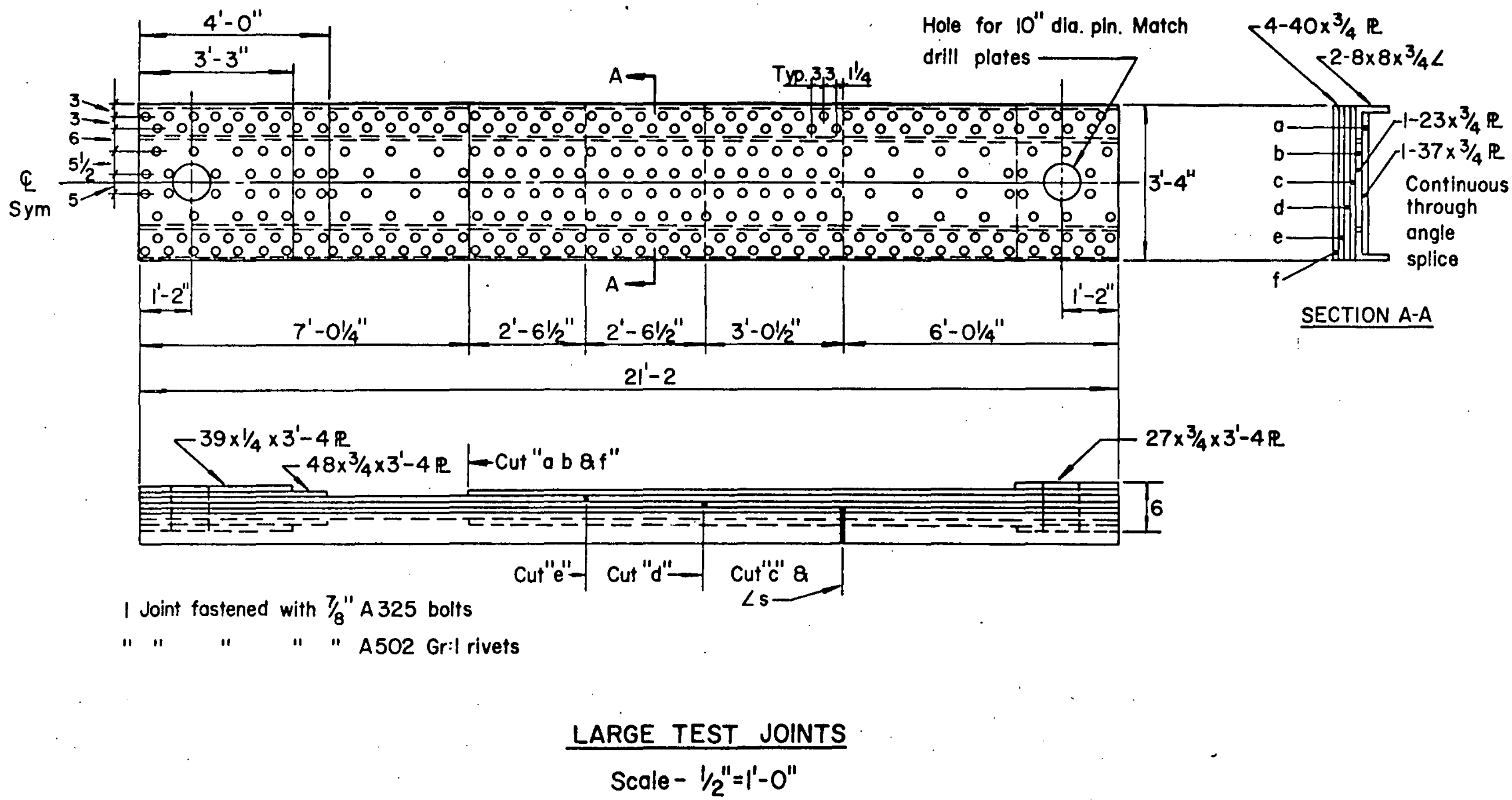

Fig. 1 Full Size Simulated Joint Test Specimen

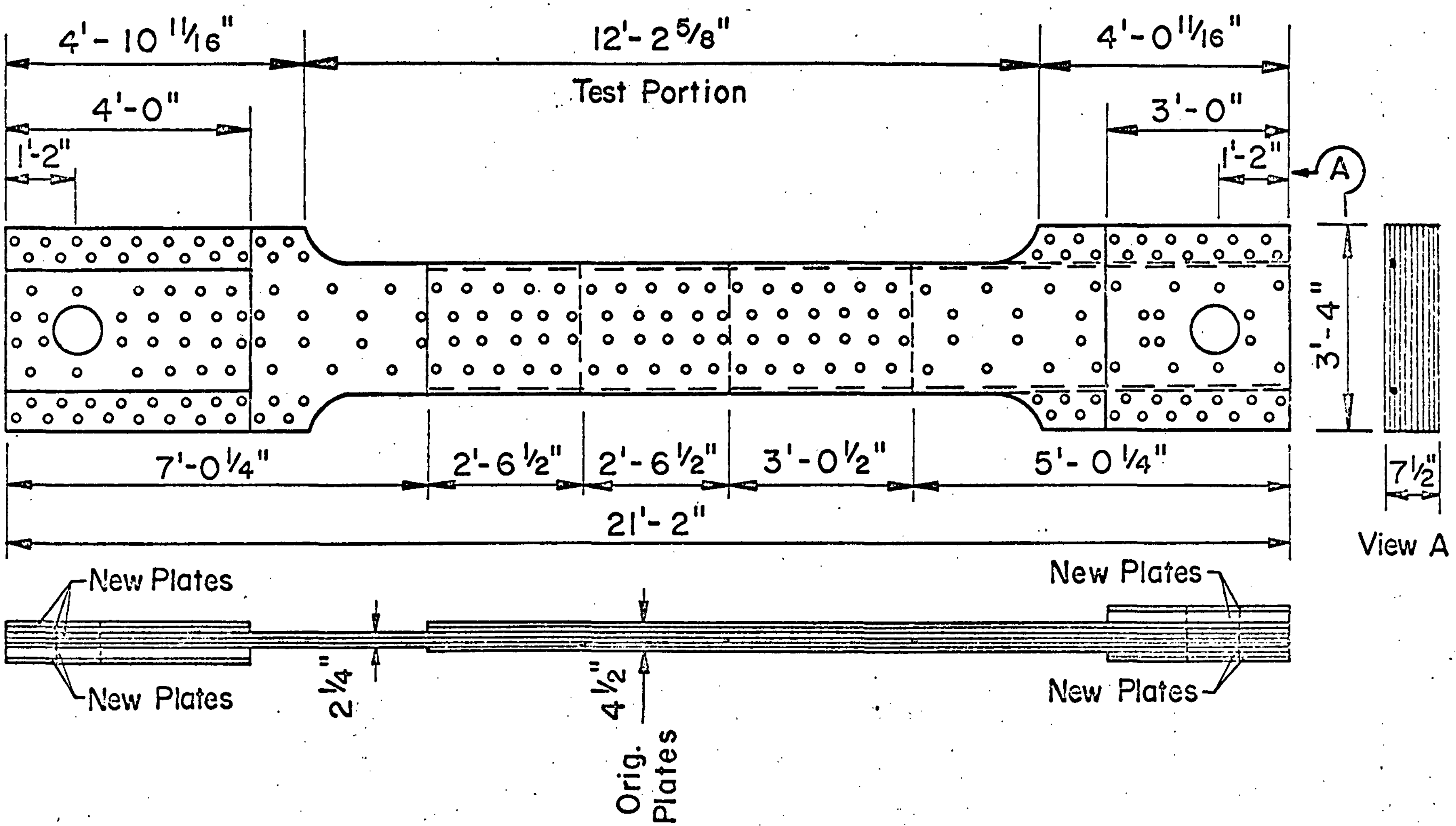
4'- 10 11/16"
12'- 2 5/8"
4'-0 11/16"
Test Portion
4'-0"
3'-0"
1'-2"
1'-2"
A
3'-4"
7'-0 1/4"
2'-6 1/2"
2'-6 1/2"
3'-0 1/2"
5'- 0 1/4"
7 1/2"
21'- 2"
View A
New Plates
New Plates
New Plates
New Plates
2 1/4"
4 1/2"
Orig. Plates

Figure 2

Shear planes

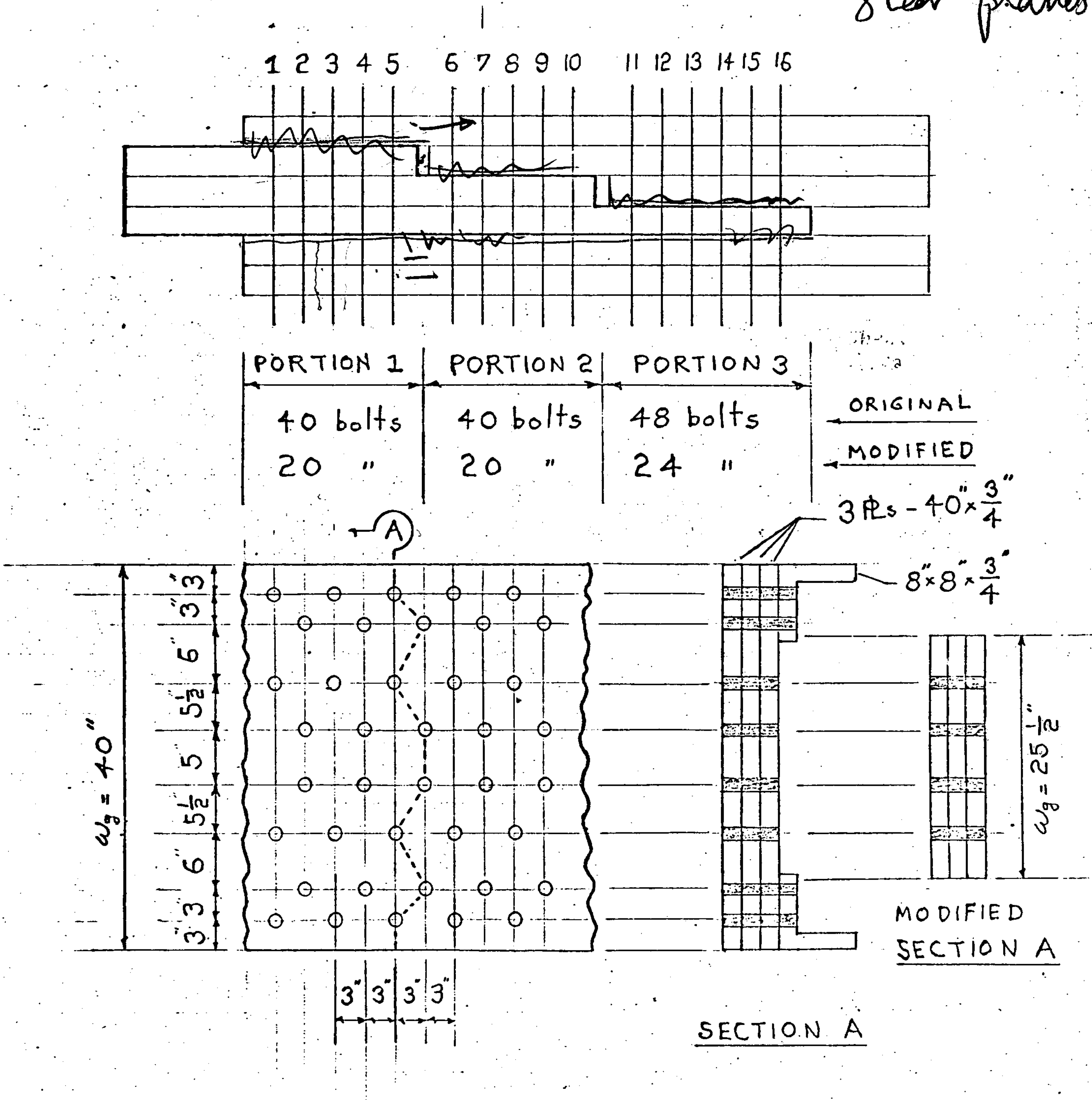

FIG. 3 SHEAR AND NET AREAS IN ORIGINAL AND MODIFIED JOINTS.

Project 357: GUIDE TO DESIGN CRITERIA
FOR MECHANICALLY FASTENED JOINTS
Sponsored by the Research Council on Riveted

Preliminary outline for the Guide has been prepared and is reproduced later in this report. Work is also currently underway in connection with the proposed revision of ASCE Manual 41 - Commentary on Plastic Design in Steel. Some material from the chapter on "Connections" is also reproduced.

8.8 DETAILS WITH REGARD TO BOLTING

Bolted joints in plastic design will probably have their greatest application as field joints for erectionof structures. The forces acting on the joint are re-

sisted by the fasteners in either tension or shear.

The shear strength of a high strength bolt is determined by the location of the shear planes. If a shear plane intersects the bolt threads, only the root area is effective in resisting the shear. The shear strength of high strength bolts was observed to be about 60% of the tensile strength.(8.17)

For high strength bolts in tension, the maximum stress is limited to the ultimate tensile strength of the steel applied to the stress area.* The ratio of the stress area to the nominal bolt area for 1/2 in to 1 in. bolts, varies from 0.725 to 0.773. Therefore, the maximum strength may be expressed as 75% of the tensile strength. Studies on both A325 andA490 bolts have shown that the tensile strength is not affected by installation.(8.18,8.19)

Nominal stress values for the factored load should ensure the ultimate strength of the member without premature failure of the connection as was suggested for welded details. A reduction factor of 0.75 is suggested to provide a further reserve strength. The reduction factor is larger than that suggested for fillet welds because mechanical fasteners do not exhibit as much variability. The resulting nominal stresses for analysis at ultimate load are:

For bolts in tension

$$\sigma_a = 0.75 \times 0.75\ \sigma_u = 0.56\ \sigma_u \tag{8.37}$$

For bolts in shear

$$\tau_a = 0.75 \times 0.60\ \sigma_u = 0.45\ \sigma_u \tag{8.38}$$

For bolts that are subjected to tension and shear, an ellipse has been fitted to the test data.(8.20) The test results for bolts with threads excluded from the shear plane and bolts with threads in the shear plane are compared with an interaction curve in Fig. 8.20.

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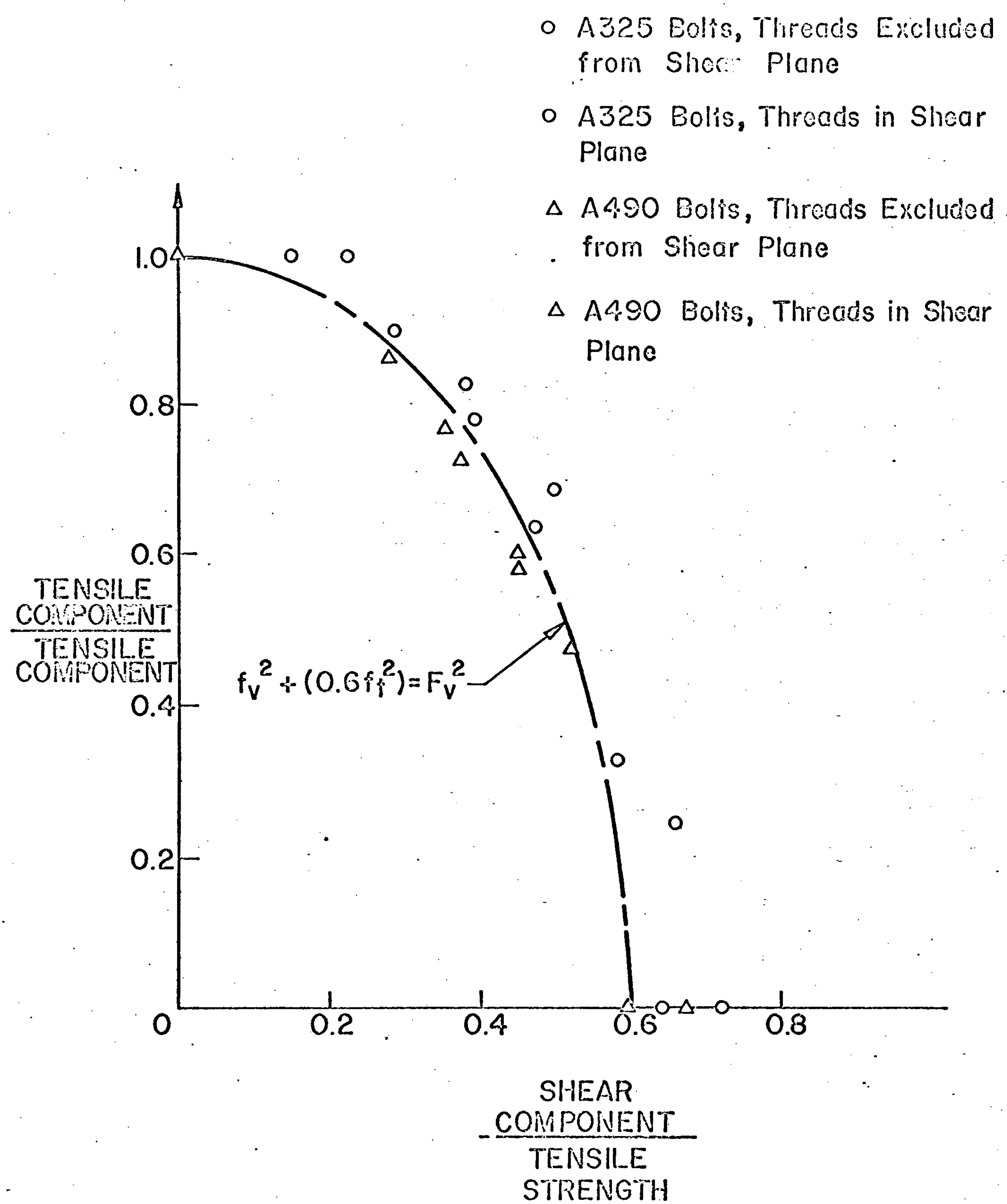
*



(8.20)

Fig. 8.20 INTERACTION CURVES

130 C

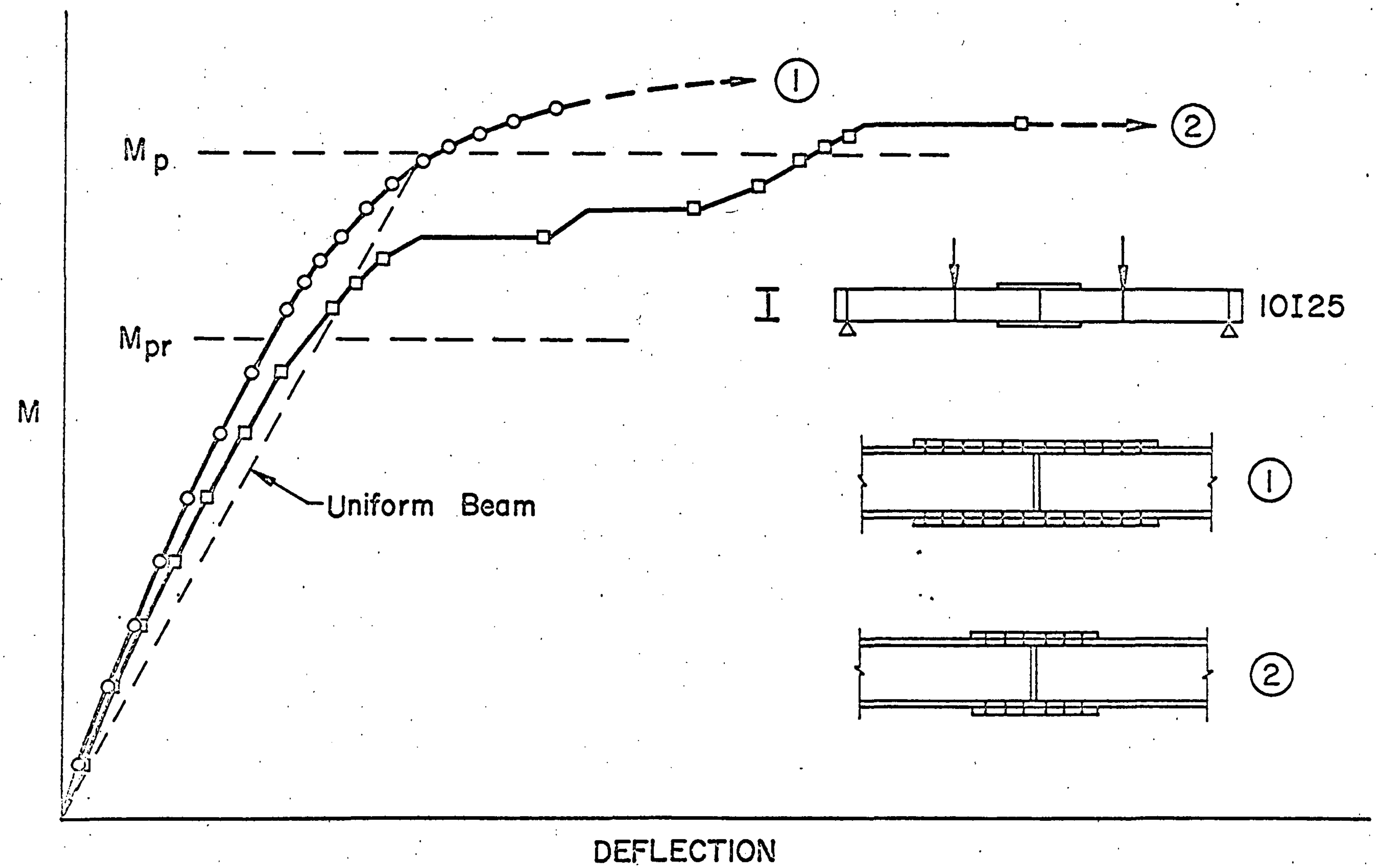


Fig. 8.21 MOMENT-DEFLECTION BEHAVIOR OF BEAM SPLICES

It is apparent that good agreement exists. Also, the root area is seen to provide an adequate measure of resistance when a shear plane intersects the bolt threads.

For bolts subjected to combined tension and shear the shear stress component should not exceed

$$f_v < \sqrt{\tau_a^2 - (0.6\ f_t)^2} \qquad (8.39)$$

where f_t = tensile stress due to the applied load.

Tests have shown that flexural members can be proportioned to resist bending on the basis of the gross cross-section as long as the holes do not deduct an excessive amount of moment capacity from the section.(8.21,8.22) Bolt holes had no appreciable effect on the test behavior even though 25% of the plastic strength was removed. This is illustrated in Fig. 8.21 which shows that M_{pr} of the net section is not the governing plastic moment. This is the result of the recognized effect of the strain-hardening.

Tests of beam splices and beam-to-column connections have shown that bolted joints can develop the full plastic moment of the connected members.(8.21, 8.22,8.23) Figure 8.22 gives the general layout of joints which are capable of developing satisfactory strength. Bolts have been subjected to shear and to tension depending on the connection type.

Bolted Splices - At maximum load bolts proportioned on the basis of 75% of their shear strength were able to develop the plastic moment. Figure 8.22 compares the behavior of beams proportioned with the bolts at 38% and 75% of their shear strength. Both were able to develop the full plastic moment. In the first case no slip developed, whereas in the second case slip occurred but did not prevent attainment

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~~An example of the assumptions that might be made in designing a high strength~~ bolted joint is given in Ref. 8.13 as exemplified by Fig. 8.19(a), 8.19(c), and 8.19(f). The number and location of bolts in a joint were first assumed. For the bolts in tension, an "ultimate" load capacity was taken in this reference as 1.3 times the proof load of the bolts. The shear resistance provided by each bolt at ultimate load due to friction was taken at one-fourth its initial tension. All structural steel parts of the joint were assumed to be stressed to yield point. To analyze the trial design, a compression area was computed sufficient ~~to resist the combined reaction of all the "working" bolts, that is, those on the~~

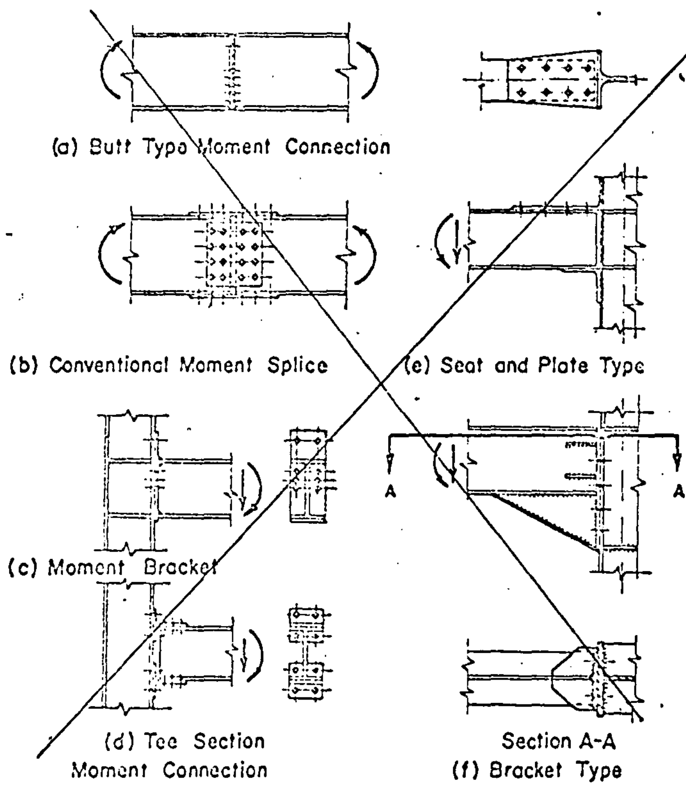
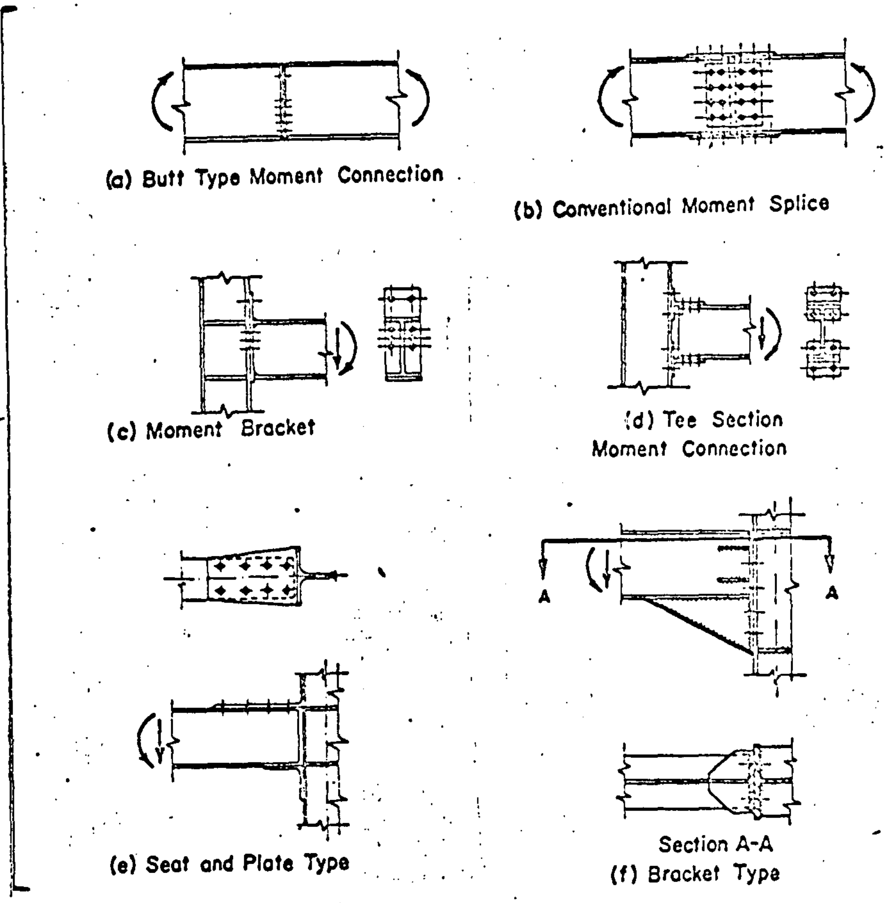


FIG. 8.19.—TYPES OF PLASTIC MOMENT CARRYING HIGH TENSILE BOLTED CONNECTIONS



8.22

~~tension side of the neutral axis (axis of rotation). The bending moment pro-~~ duced by the couple made up of the bolt tension forces and the compression force, had to equal or exceed the applied bending moment.

Ref. 8.13 notes that plastically designed joints with high tensile bolts in tension require a smaller number of bolts and less fitting material than conventional moment splices which use the bolts only in shear.

Although accurate procedures leading to the most economical safe design are not yet available, the results of research show that safe bolted joints can be designed to develop the plastic moment of the members with reasonable ~~economy.~~

of the plastic moment. M_{pr} in this figure is ~~with tl holes removed~~. the plastic moment of the net section.

The suggested design value of 0.45 σ_u provided satisfactory connection behavior as illustrated by beam 2 in Fig. 8.21. The beam was able to develop its full plastic moment capacity. Also, note that slip had no adverse effect on the member behavior. In most cases it will provide a beneficial effect because it will permit a more favorable distribution of moment and does not require as much rotation capacity.

Web splices have resisted the applied shear without adverse affect.[8.22,8.24]

When the splice length exceeds 35 inches, the reduction in strength due to the joint length does not provide the desired reserve capacity. Figure 8.23 illustrates this behavior for shear splices.[8.25] It is unlikely that the splice ~~lenght~~ will exceed 35 inches in buildings. Hence, the suggested design value ensures an adequate margin of strength in the connection so that the ultimate load can be developed.

Beam splices[8.21,8.22] as illustrated in Fig. 8.22b, rigid frame corner connections[8.28] as illustrated in Fig. 8.24 and splice plates have all demonstrated the ability to develop the desired strength when the fasteners have been proportioned to resist the plastic moment at the suggested design levels.

T-Stubs Flange-to Column Connections - A considerable amount of work has been undertaken during the past ten years on beam-to-column connections fastened with high-strength bolts.[8.21,8.22,8.23] A commonly used connection incorporates a T-stub to transfer the beam moments into the column. On the beam compression flange, the T-stub acts much like a bearing pad so that only column web crippling or crippling of the T-stub web can occur. Equation 8.21

can conservatively be used to evaluate the adequateness of the column web for this situation. The T-stub connecting the beam tension flange is more critical. Particular attention must be paid to the tensile forces in the bolts connecting the T-stub flange to the column face. Because of distortion of the column or T-stub flanges prying forces may develop.

In Fig. 8.25 are shown schematically the forces and distortions of a simple T-stub. Under zero applied load the tension in each bolt if T_o, the initial clamping force. When the external load is applied, the flange deflects causing prying forces to develop, Reference 8.22 developed a semi-empirical estimate of the prying force Q as follows

$$Q = \left[\frac{1/2 - wt^4/30ab^2A_b}{a/b\left(a/_{3b} + 1\right)wt^4/6ab^2A_b} \right] F \qquad (8.40)$$

in which

- Q = Prying force
- w = length of T stub flange
- a = distance from center of bolt to edge of plate
- b = distance from center of bolt to center of fillet of connected part
- A_b = Nominal bolt area
- t = thickness of T-stub flange
- F = average force per bolt (T/n)

Figure 8.26 compares the observed bolt force with the computed tension based on Eq. 8.40. It is seen that Eq. 8.40 gives a reasonable agreement with the test results. Almost all measure bolt tensions in the stub-to-column bolts of tests reported in Ref. 8.22 were equal or less than computed values.

Because of the complexity of the analytical solution given by Eq. 8.40 various variables were evaluated to establish their significance. This study showed that prying action could be approximated with reasonable

131c

accuracy using the following equation.

$$Q = \left[\frac{3b}{8a} - \frac{t^3}{20} \right] F \qquad (8.41)$$

A comparison of the simplified expression given by Eq. 8.41 with Eq. 8.40 is given in Fig. 8.27. The approximation is seen to provide a conservative estimate of the prying force for all bolt diameters.

In arriving at a simple analysis of the T-stub flange, the approach suggested by Schutz[8.23] seems reasonable. The equations for the required thickness can be determined by considering plastic hinges to form in the flange as shown in Fig. 8.28. The force in the flange is given by

$$T \approx \frac{M_p}{d_b} \qquad (8.42)$$

in which d_b = the beam depth.

Now, the plastic hinges forming in the flange adjacent to the web and bolts have values of

$$M_{pw} = \frac{wt^2\sigma_y}{4} \quad \text{and} \quad M_{pb} = \frac{kwt^2\sigma_y}{4} \qquad (8.43)$$

where k provides for the presence of the bolt holes. Tests reported in Ref. 8.22 have shown that k can be taken as 1 because strain hardening enables the flanges to resist a continuously increasing load. Failure ultimately occurs by fracture of the T-web or a bolt. Thus

$$M_{pw} = M_{pb} = M_{pt} \qquad (8.44)$$

The simple mechanism that is formed enables one to evaluate the required flange thickness

$$M_{pw} + M_{pb} \approx \frac{Tb}{2} \qquad (8.45)$$

therefore

$$\frac{2wt^2\,\sigma_y}{4} = \frac{M_{pt}b}{2d_b}$$

$$t = \sqrt{\frac{M_{pt}b}{wd_b\sigma_y}} \tag{8.46}$$

Equation 8.46 can be used to make an initial selection of the T-stub. However, it is more likely that prying section will control so that it may be necessary to increase the flange thickness if it is necessary to minimize the prying effect given by Eq. 8.41.

End Plate Connections - End plates welded to beam ends were suggested in Refs. 8.21, 8.23 and 8.24, for a variety of types of connections. For example moment brackets, butt-type beam splices and beam-to-column connections (See Fig. 8.21) were tested to provide an indication of their behavior and performance.

These studies have all indicated that the shear force can be readily transmitted by the friction developed between the end plate and the element it is connected to. Its resistance to slip can be evaluated in terms of the bolt preload and the nominal slip coefficient.

The tests have also indicated that when end plates extend beyond the tension flange as shown in Fig. 8.21c that the bolts which are chiefly effective in resisting the tension flange force are those adjacent to the tension flange. References 8.22 and 8.23 have suggested that the bolts and the part of end plate at the tension flance can be treated as an equivalent T-stub connection. Equations 8.41 and 8.46 can be used to evaluate the adequacy of the T-stub and the bolt forces. For large beams additional bolts can be used to develop the tension in the inner portions of the web. The test results have indicated that connections designed on this basis will provide adequate strength.

131e

BOLT STRESSES

Bolts in Tension:

$$\sigma_a = 0.56\ \sigma_u \tag{8.37}$$

where σ_u is the tensile strength of the bolt. The bolt area is taken as the nominal area.

Bolts in Shear:

$$\tau_a = 0.45\ \sigma_u \tag{8.38}$$

where σ_u is the tensile strength of the bolt. The bolt shear area is taken as the nominal area when the shear plane intersects the shank. The root area is used when the shear plane intersects the threads.

For combined tension and shear bolts shall be proportioned so that the shear stress does not exceed

$$f_v < \sqrt{\tau_a^2 - (0.6\ f_t)^2} \tag{8.39}$$

where f_t = tensile stress due to the applied load.

The tension due to prying action shall be computed as

$$Q = \left[\frac{3b}{8a} - \frac{t^3}{20}\right] F \tag{8.41}$$

BOLTED BEAM-TO-COLUMN CONNECTIONS

The provisions of Article 8.6 apply for column stiffeners and for shear stifening of the columns.

T-Stub Flange

The minimum thickness of T-stub flanges is

$$\tau = \sqrt{\frac{M_{pt}\ b}{wd_b\ \sigma_y}} \tag{8.46}$$

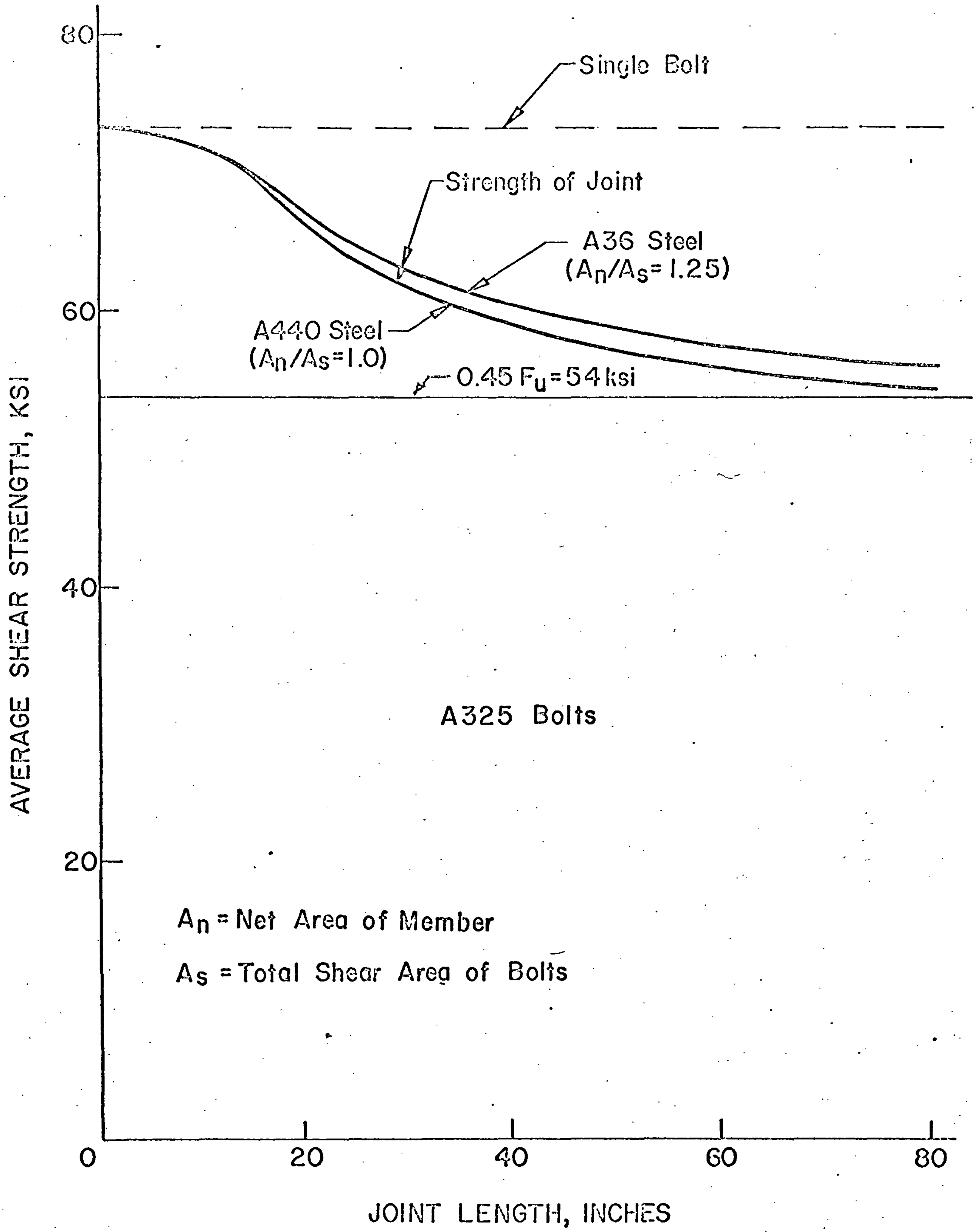


Fig. 8.23 EFFECT OF LENGTH ON THE SHEAR STRENGTH OF SPLICES

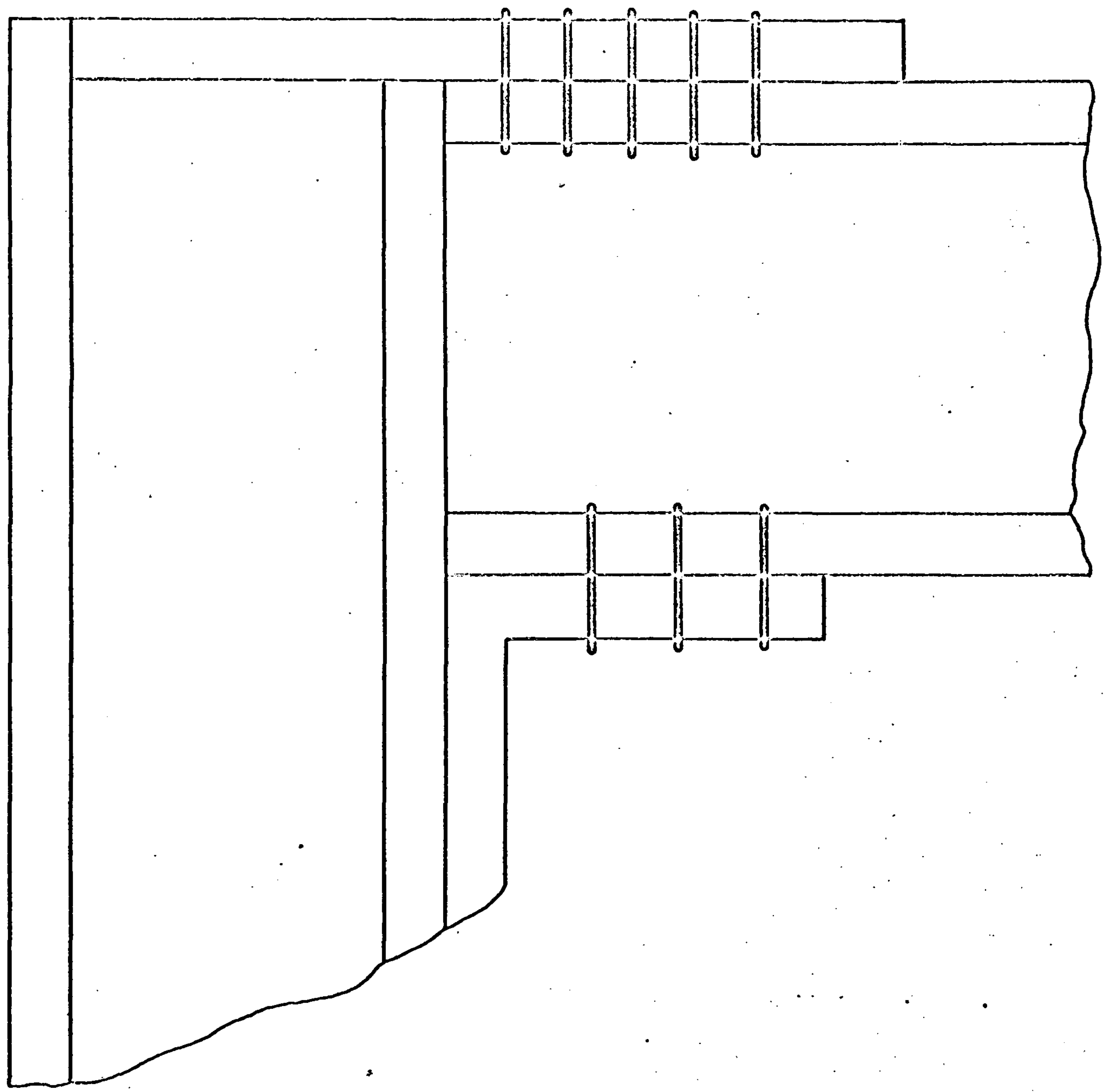
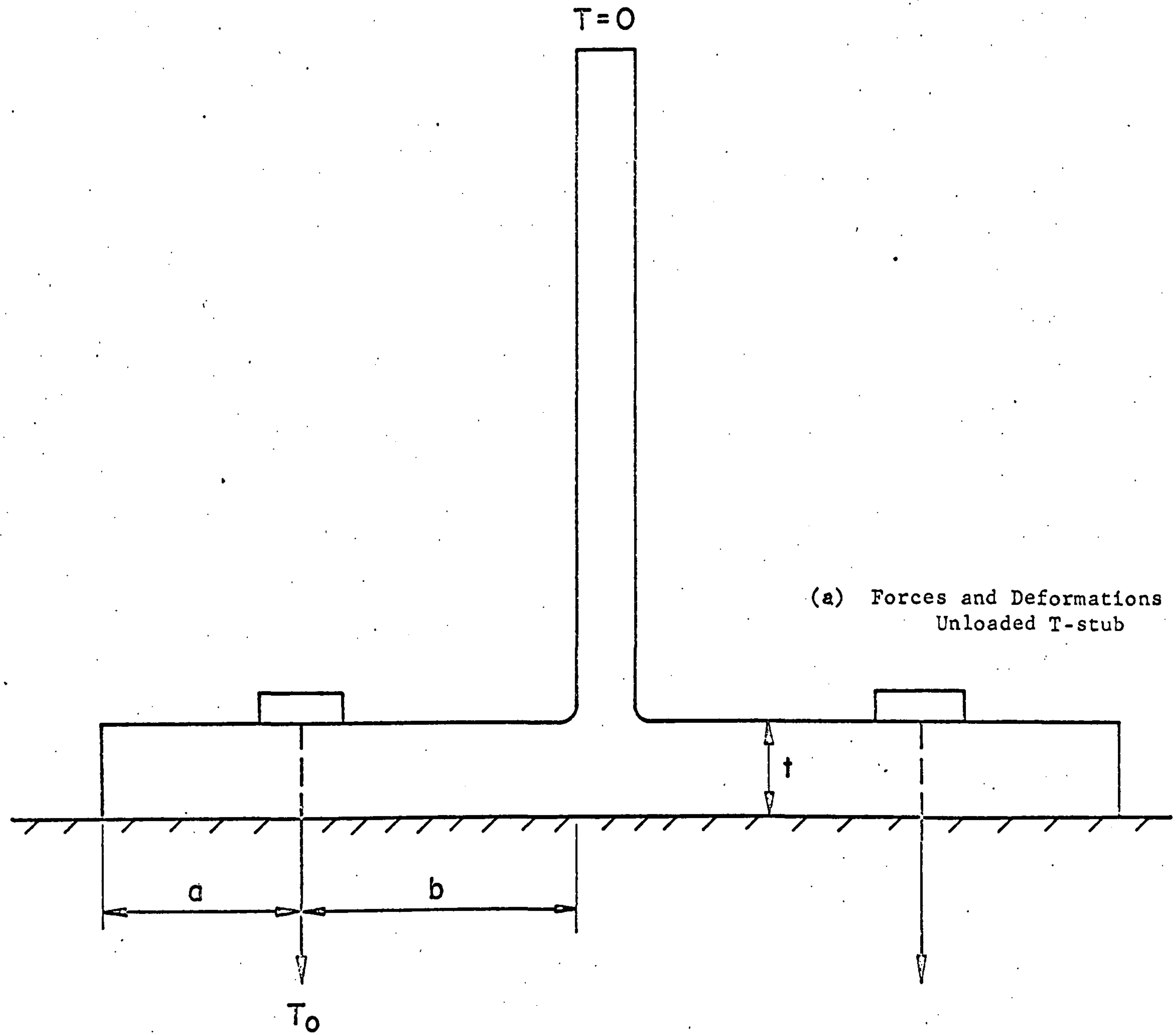
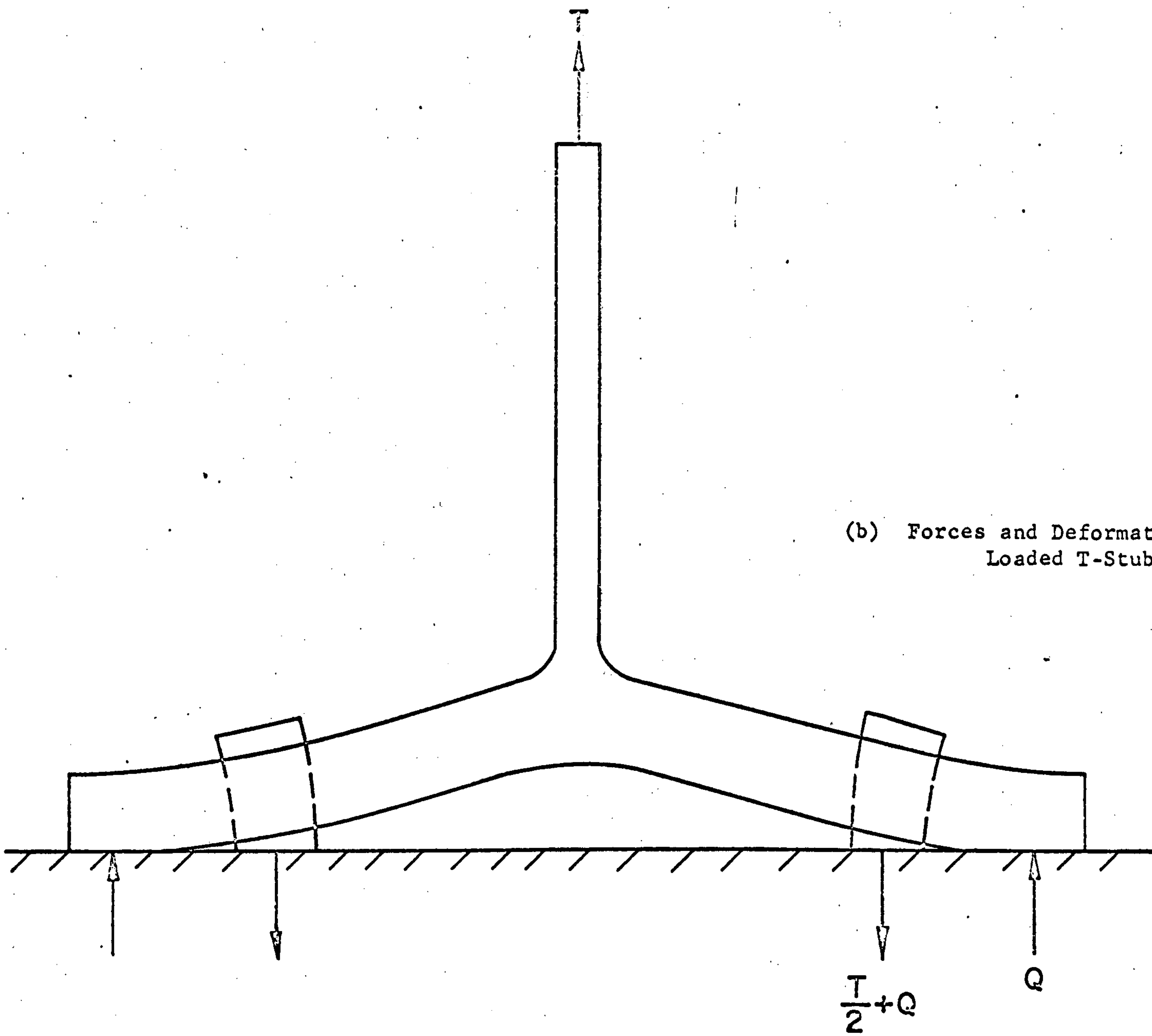


Fig. 8.24 RIGID FRAME CORNER CONNECTION



(a) Forces and Deformations in Unloaded T-stub

Fig. 8.25 ASSUMED FORCES AND DEFORMATIONS IN T-STUBS



(b) Forces and Deformations in Loaded T-Stub

Fig. 8.25 ASSUMED FORCES AND DEFORMATIONS IN T-STUBS

131 c

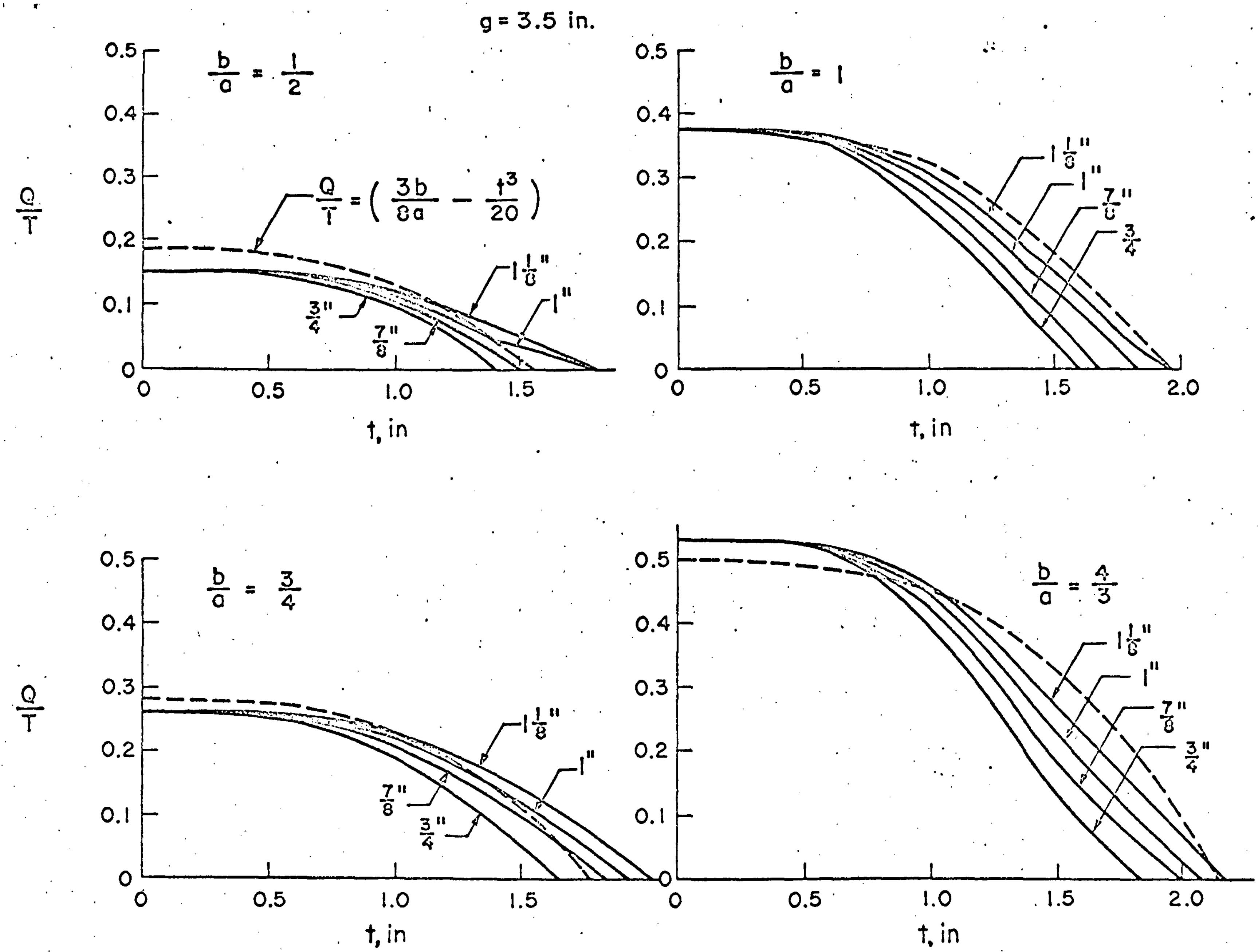


Fig. ~~A-19~~ – Comparison of design recommendations with semi-empirical formula

8.27

38 12/72

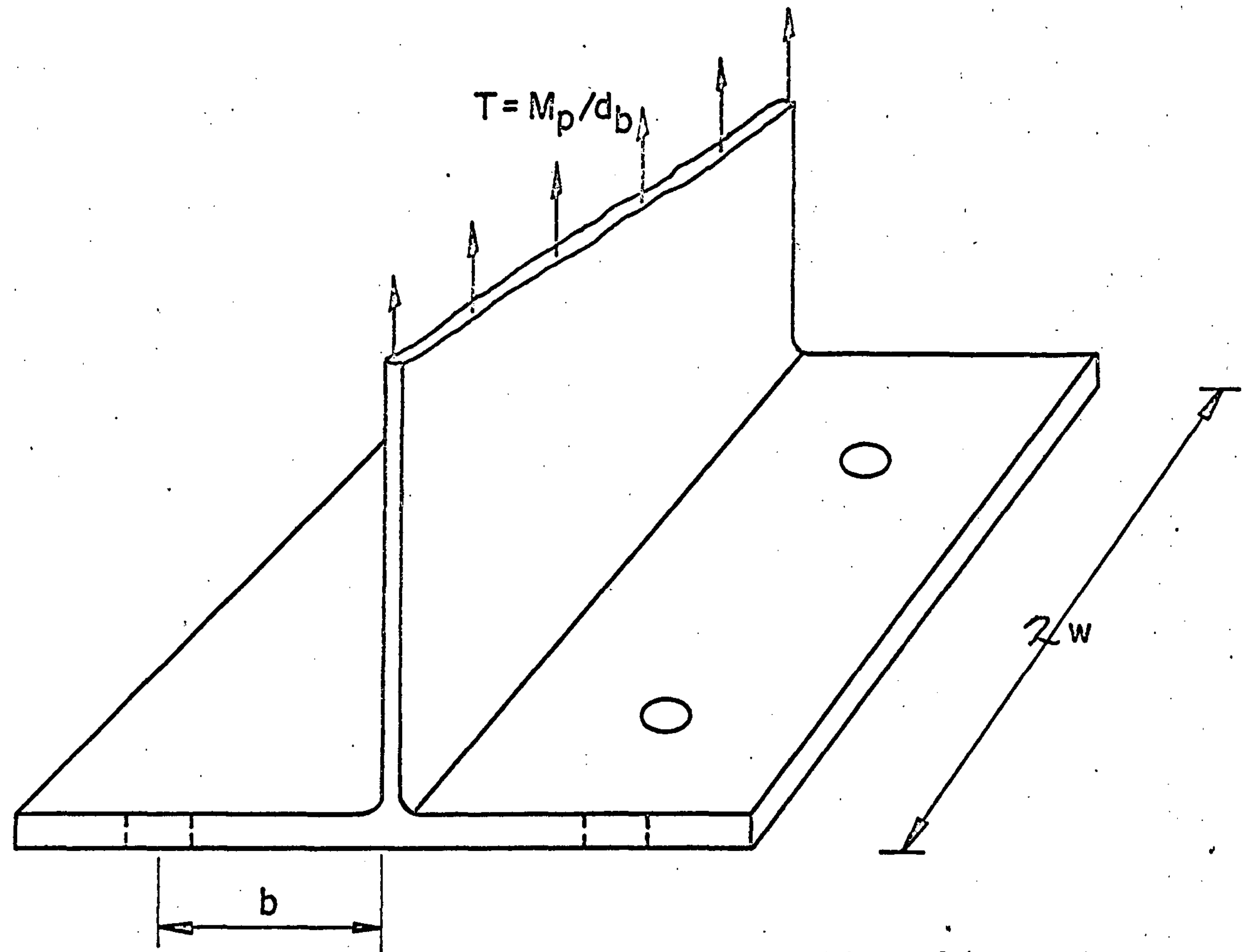
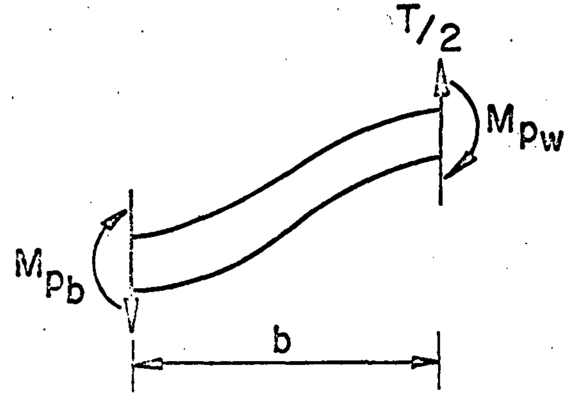


Fig. 8.28 SYSTEM OF FORCES ACTING ON TEE FLANGES



131-2

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